TOWN OF LEDUC
GENERAL MUNICIPAL PLAN
WATERWORKS & SEWERAGE

SSOCIATED ENGINEERING SERVICES LTD

Thirty-Five Years Of Engineering Excellence
13140 - ST. ALBERT TRAIL, EDMONTON, ALBERTA, T5L 4R8. TEL. (403) 453-8111. TELEX: 037-2333

April 24, 1981
File: EM45

Mayor and Members of Council
Town of Leduc
Leduc, Alberta
T9E 2Y1

Dear Sirs:

We are pleased to submit 50 copies of our report entitled "General Municipal Plan - Waterworks & Sewerage".

The report was prepared in conjunction with the General Municipal Plan and examines waterworks, sanitary sewerage and storm drainage requirements. The proposed system, based on the design criteria established in the report, will provide adequate services at ultimate development. Therefore, we recommend to Council that this report be adopted as a "Master Plan" for the purpose of planning major municipal utilities within the General Municipal Plan Area.

We are pleased to have had the opportunity to work with you on this project and would be pleased to meet with Council to discuss further any aspect of the report.

Respectfully submitted

A.E.S.L.

Grant Bridgeman

G. Bridgeman, P.Eng.

Encls.

GB/gc

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I. INTRODUCTION

PREFACE

The "municipal services" examined herein comprise of the waterworks, sanitary sewer and storm drainage systems. This report was prepared in conjunction with the Town's General Municipal Plan; therefore, it should be considered as a supplementary engineering analysis to the General Plan prepared by the Edmonton Regional Planning Commission.

The Town of Leduc on November 5, 1979 authorized Associated Engineering Services Ltd. to undertake an engineering study for purposes of planning the long range needs of municipal services (waterworks, sanitary sewerage and drainage facilities) required for the General Municipal Plan Area.

The purpose of the engineering analysis is to provide cost efficient municipal services for the expansion area established by the General Municipal Plan. The results of this study are outlined in the various chapters that follow.

METHODOLOGY

The engineering analysis was carried out in two phases.

Phase 1

A preliminary analysis was carried out for four expansion alternatives including the assessment of the existing facilities, establishment of required facilities to serve the various expansion alternatives and the feasibility cost analysis to determine the most economical expansion alternative.

Phase 2

In the final phase of the engineering analysis design criteria used in Phase 1 were reviewed and established for final analysis; system components were analyzed and established for implementation; a "master plan" for each municipal service (water, sanitary sewer and drainage) was prepared for future implementation purposes and recommendations were made for improvements required in the near future with order of magnitude costs in 1981 dollars.

It must be recognized that the proposed servicing systems presented in this study are conceptual only and may change prior to Development of the affected Areas. Therefore, the review of proposed facilities is required prior to implementation based on detailed land use and planning requirements.

ACKNOWLEDGEMENTS

Associated Engineering Services Ltd. gratefully acknowledges the advice and assistance provided by the members of the General Municipal Plan Committee, the Town's Administration and Engineering staff and the General Plan staff of the Edmonton Regional Planning Commission.

REFERENCES

1. Town of Leduc - Water Storage and Distribution
prepared by AESL, August 1972.

2. Strathcona-Leduc Water Supply System Pumphouse Modifications
prepared by AESL, November 1972.

3. Town of Leduc - Infiltration System
prepared by AESL, January 1975

4. Town of Leduc - Waterworks and Sewerage Study
prepared by AESL, Janury 1975

5. Strathcona-Leduc Water Board New Water Supply Pipeline Concept Report
prepared by AESL, July 1978

6. Edmonton Regional Utilities Study
prepared by Alberta Environment, July 1978.

7. Nisku General Development Plan
prepared by Stewart, Weir, Stewart, Watson, Heinrichs & Dixon; and AESL, February 1979.

8. Strathcona-Leduc Water Board Water Supply Pipeline Operating Manual
prepared by AESL, 1979.

9. South East Regional Trunk Sewerage System
prepared by Stanley Associates Engineering Ltd, September 1979.

10. A Preliminary Assessment of Expansion Alternatives for the Town of Leduc prepared by the Edmonton Regional Planning Commission, January 1980.

11. Town of Leduc Preliminary Sewer System Infiltration Analysis
prepared by Mar-Tech Engineering Co. Ltd., April 1980.

12. Town of Leduc Draft General Municipal Plan prepared by the Edmonton Regional Planning Commission in 1981.

LIST OF ABBREVIATIONS

Metric Units

ha	-	hectare
m^3	-	cubic metres
m^3/s	-	cubic metres per second
m^3/d	-	cubic metres per day
L/d	-	litres per day
L/c/d	-	litres per capita day
L/ha/d	-	litres per hectare per day
L/s	-	litres per second
L	-	litres
m	-	metres
mm	-	millimetres
m/s	-	metres per second
kPa	-	kiloPascal
km	-	kilometres

Imperial Units

ac	-	acre
ac ft	-	acre feet
mig	-	million imperial gallons
migd	-	million imperial gallons per day
igpd	-	imperial gallons per day
igpm	-	imperial gallons per minute
igpcd	-	imperial gallons per capita day
igpd/ac	-	imperial gallons per day per acre
cfs	-	cubic feet per second
ig	-	imperial gallons
ft	-	feet
in	-	inch
fps	-	feet per second
cu ft	-	cubic feet
psi	-	pounds per square inch

Miscellaneous

TDH - Total Dynamic Head

AESL - Associated Engineering Services Ltd.

ERPC - Edmonton Regional Planning Commission

II. DESIGN PARAMETERS

GENERAL

The assessment of existing facilities and establishment of required facilities are based on design criteria established in Appendices A, B, C and D.

DESIGN PERIOD

Water trunk mains, sanitary trunk sewers, storm trunk sewers and storm retention facilities in this report are sized to serve their tributary areas at ultimate development. However, for reservoirs and pumping facilities a period of 10 years is used as the capacity of these facilities can be readily increased to meet projected population growth.

LAND USE AND POPULATION PROJECTION

Land use areas for various purposes and population projection for year 1991, 2001 and ultimate development were established in the General Municipal Plan and were two basic design criteria used in this Study.

Map 1 - Geographic Location, Map 2 - Existing Land Use 1979 and Map 3 - Future Urban Form are included in the Draft General Municipal Plan. Since these maps form an integral part of the General Municipal Plan they were not included in this report.

TOPOGRAPHIC INFORMATION

The accuracy of any servicing concept is dependent on the accuracy of the contour plan available for the Study Area. In the Engineering Analysis a 1 m contour plan was used, which provides complete coverage within the northern, southern, eastern and western limits of the present boundaries. Beyond these limits; however, the 1 m contour plan is not available, and a 10 foot contour plan of the National Topographic Series was used. Therefore, the accuracy of analysis south and west of the present boundaries is limited to the accuracy of the contour plan used.

III. WATERWORKS SYSTEM

GENERAL

In this report "Waterworks System" refers to the system of reservoir, mains and facilities by which water is obtained and distributed.

DESIGN CRITERIA

Detail design criteria used to evaluate the existing waterworks system and in the preliminary design of proposed and future facilities are outlined in Appendix A.

WATER SUPPLY

Water for the Town of Leduc is conveyed from the City of Edmonton waterworks system by the Strathcona-Leduc Water Board's new 600 mm (24-in.) diameter steel supply line. The construction of this supply line was completed in 1979.

The new pipeline was designed as a transmission main and sized to supply water to the year 2001 at the peak day demand rate (2 x average day). This means that communities connected to the supply line, including the Town of Leduc, must provide facilities to meet their peak water demands such as the peak hourly supplement and fire flows.

The Water Board's projected peak day water demand for the Town of Leduc is 421 L/s (5555 igpm) at year 2001. This projection is based on the design criteria of 455 L/c/d (100 igpcd) composite water demand established in the Edmonton Regional Utilities Study.

The Town's projected 2001 peak day water demand in this report is 574 L/s (7575 igpm) based on the projected population growth and anticipated commercial and industrial developments. The discrepancy between the two projections results from the criteria for the water demand per day per acre for industrial areas. This is due to inadequate information regarding the type and composition of future industrial developments and the average per acre water demands required to adequately serve future industrial development areas.

Therefore, the rate of peak day water demands should be reviewed and reassessed at least every 10 years and peak day demand requirements be adjusted to reflect actual field conditions. To achieve this a systematic metering and recording of water consumption for the various land use areas should be established and maintained by the Town on a continuous basis.

The Town of Leduc presently receives water at distribution system pressure from the Water Board. With the proposed operating philosophy of the Water Supply System (discharging water at zero pressure to the individual customer's reservoir), major operating changes will be required at the supply point to the Town of Leduc.

Presently there is no reservoir or repumping facilities existing at the supply point to the Town of Leduc. Therefore, to achieve the proposed operating philosophy of the Water Board, supply reservoir and repumping facilities will be required in north Leduc at a point where the supply line crosses Highway 2 as shown on Map 4. Stage 1 of these facilities should be adequate to meet 1991 peak day demands and supply storage requirements as shown in Table A-1 and Table A-2 in Appendix A.

WATER DISTRIBUTION SYSTEM

General

The function of the distribution system is to transport water to consumers, fire hydrants, and sprinkler systems located throughout the Town. Pipes should be adequately sized to carry the volumes of water required for the area it serves and still retain adequate residual pressure at the terminal points satisfactory for consumer use and fire fighting purposes.

The feeder main system should be flexible so extensions into future development areas can be achieved economically and with minimum effect on the existing distribution system. This is an important factor for systems that serve a rapidly expanding community.

Operating Pressure

Pressure in the system is considered adequate when it is above 276 kPa (40 psi) during peak hourly water demand periods at any point in the system or when it is at least 138 kPa (20 psi) at the point of fire and 276 kPa (40 psi) elsewhere in the system during simultaneous fire flow and peak day demand periods. Pressure between 138 kPa (20 psi) and 276 kPa (40 psi) however is acceptable only in the immediate vicinity of the fire.

Presently, adequate pressure in the distribution system is maintained by the existing water supply line, elevated tank and high lift pumps in the distribution reservoir. In the future, when the operation of the supply line changes and repumping at the master meter is required, operating pressure at the master meter should be maintained at 490 kPa (71 psi) until year 2001 and 524 kPa (76 psi) after year 2001. Pressure at the distribution reservoir should be increased to 427 kPa (62 psi) at ultimate development.

Existing Water Distribution and Storage Facilities

The original water distribution system was installed in 1953 and was upgraded gradually since early 1970. The present system consist of major feeder mains from 600 mm (24-in.) to 300 mm (12-in.) diameter as shown on Map 4 to 6. These major water feeders are cross-connected with distribution mains from 150 mm (6-in.) to 250 mm (10-in.) diameter to meet the various water demand at required pressures.

Until 1974 water storage for the Town was provided by the existing 182 m^3 (40,000 ig) elevated tank. In 1974 the waterworks system was strengthened by a 6,819 m^3 (1.5 mig) underground reservoir and pumping facilities in Corinthia Park. The purpose of these facilities is to store water during low water demand periods and to supplement the capacity of the supply line during peak water demand periods such as peak hour and fire flow. The Corinthia Park reservoir (distribution reservoir) and pumphouse was designed to accommodate future expansion on the same site to keep pace with growth and development of the Town.

Based on the design criteria established in Appendix A the performance of the existing waterworks system is adequate to serve the existing population.

Proposed Water Distribution System

To solve the many problems that arise in planning and operating the water distribution system and to establish the most economical system to serve all areas within the study boundaries at ultimate development, a computerized hydraulic network analysis was used.

The existing and proposed feeder mains were analyzed for the following water demand conditions at ultimate development:

1. Peak Hour Demands
2. Peak Day Demand plus Commercial Fire Flow.
3. Peak Day Demand plus Industrial Fire Flow.
4. Peak Day Demand plus Residential Fire Flow.
5. Peak Week Demand plus Filling Reservoir.

The existing and proposed feeder mains and pressure contours for the first three water demand conditions are shown on Map 4 to 6.

Although Item 4 and 5 water demands are not considered critical, pressure under peak week demand and filling reservoir condition drops to 214 kPa (31 psi) at the most southeasterly part of the system. Since reservoir fill normally occurs during low water demand periods this occasional low pressure condition at the south-east may be considered tolerable or development in this area could be restricted to single family development.

Reservoir Pumping Facilities

The existing reservoir pumphouse is equipped with two 44 L/s (580 igpm) service pumps and a 221 L/s (2917 igpm) fire pump. All pumps are discharging against a head of 36.6 m (120 feet). Presently the service pumps capacity in parallel operation is adequate. The capacity of the fire pump is also adequate until the maximum fire flow rate is maintained at 265 L/s (3500 igpm). Additional service pump capacity; however, will be required to meet the peak hour supplement at year 1991.

The network analysis shows that the pressure at the reservoir and at ultimate development should be increased to 427 kPa (62 psi) from the present 359 kPa (52 psi). Since the present head of the pumps will be adequate in the next five to ten years the following policy should be adopted for increasing the head of the pumps:

1. Any new service pumps should be installed with a minimum head of 44 m (144 feet) or 427 kPa (62 psi).

2. When the existing pumps require servicing the existing type "B" bowl assembly should be replaced with type "A" bowl assembly. Type A impellers should be checked to ensure utilization of the existing pump drive.

Water Storage

The capacity of the Town's existing distribution reservoir is 6819 m^3 (1.5 mig). This is slightly less than the computed capacity of 7746 m^3 (1.704 mig) required for the present development conditions. To meet the 1991 requirements for distribution storage an additional 6819 m^3 (1.5 mig) storage volume should be added in 1981. This addition would increase the total storage to 13638 m^3 (3.0 mig) which would be 2308 m^3 (0.508 mig) less than the 1991 requirements of 15946 m^3 (3.508 mig) as shown in Table A-2 in Appendix A.

Proposed Improvements

Based on the design criteria shown in Appendix A and the results of the hydraulic network analysis, it was concluded that the existing waterworks system (water supply, water distribution system, water storage and pumping facilities) are adequate to meet the Town's present water demands at the desirable minimum pressure. However, to increase the reliability of the waterworks system and to keep pace with development within the General Plan Study Area the capacity of reservoirs and pumping facilities must be increased to meet water demand requirements at year 1991. To achieve this the following improvements are required:

1. A 13638 m^3 (3.0 mig) supply reservoir complete with repumping facilities at the water supply point (north boundary) of the Town.

2. An additional 6819 m^3 (1.5 mig) reservoir cell at the Corinthia Park reservoir and one additional 102 L/s (1,350 igpm) service pump with 44 m (144 ft) total dynamic head (TDH).

3. After the supply reservoir and repumping facilities are completed the system pressure control should be relocated at the Corinthia Park reservoir and the elevated tank should be taken out of operation.

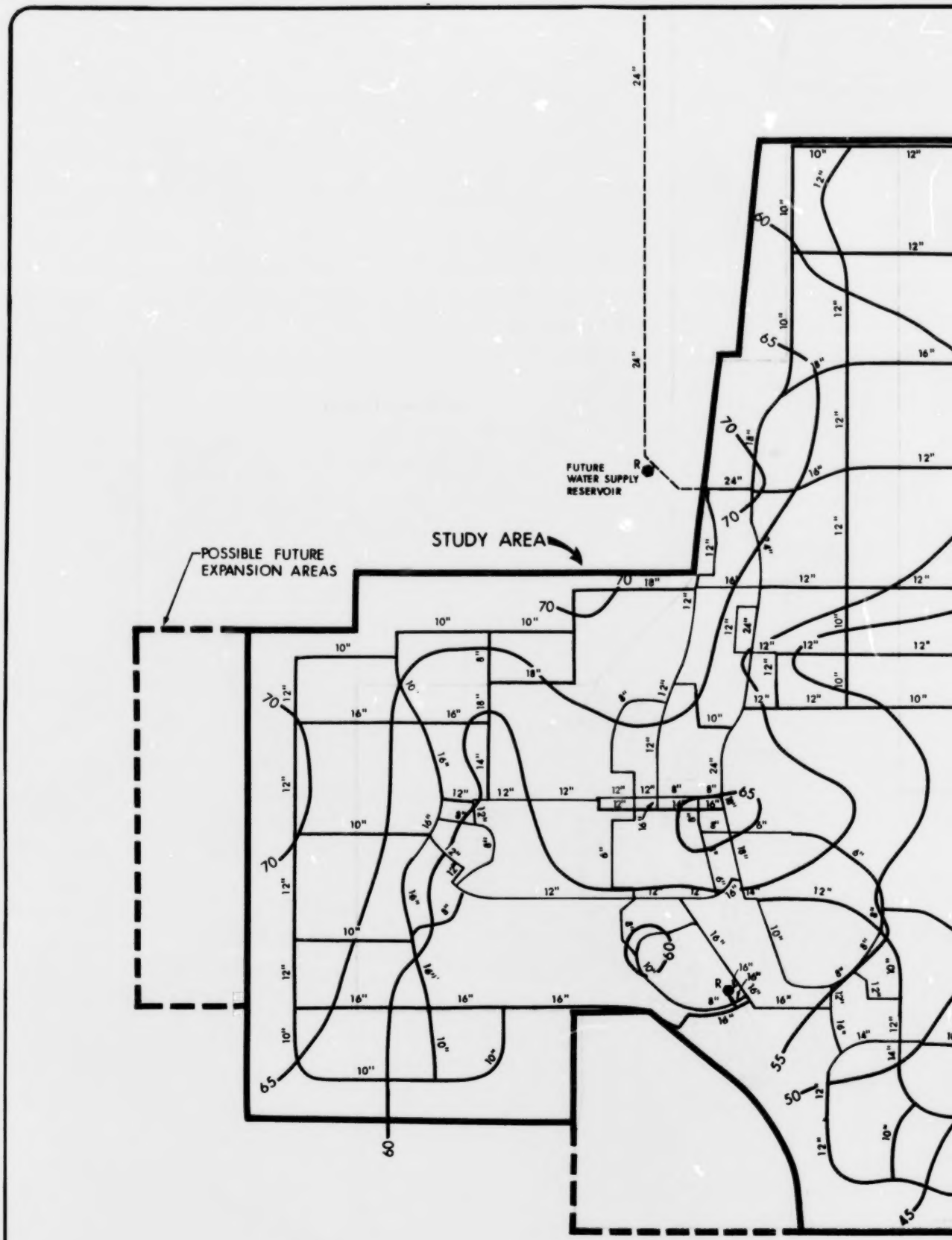
4. Water feeder mains should be implemented as required by population growth and land development.

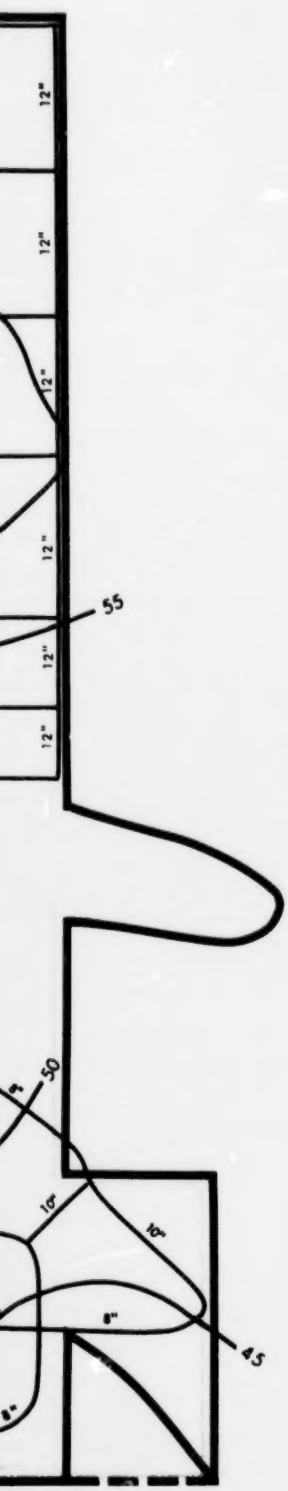
Cost Estimate of Proposed Improvements

The estimated cost of the required improvements in 1981 dollars are as follows:

1.	A 13,638 m^3 (3.0 mig) underground reservoir and pumping facilities at the water supply point	$3,800,000
2.	A 6,819 m^3 (1.5 mig) additional cell at the Corinthia reservoir and installation of one service pump	$1,200,000
	TOTAL	$5,000,000

The above cost include an allowance for engineering and contingencies.


FUTURE
WATER SUPPLY
RESERVOIR
R
STUDY AREA
POSSIBLE FUTURE
EXPANSION AREAS



LEGEND

12" EXISTING TRUNK MAIN

EXISTING SUPPLY LINE

R EXISTING RESERVOIR

16" PROPOSED TRUNK MAIN

60 60 PRESSURE CONTOUR IN POUNDS PER SQUARE INCH

PIPE DIAMETERS:

SI UNITS mm	IMP UNITS inches
200	8
250	10
300	12
350	14
400	16
450	18
600	24

NOTE: ONLY WATERMAINS INCLUDED IN ANALYSIS ARE SHOWN.

PRESSURE CONTOURS:

KILOPASCALS	POUNDS PER SQUARE INCH
172	25
207	30
241	35
276	40
310	45
345	50
379	55
414	60
448	65
483	70
517	75

ASSOCIATED ENGINEERING SERVICES LTD AESL

N

kilometers 0 .5 1 1.5 2

miles 0 .5 1 1.5

Edmonton Regional Planning Commission

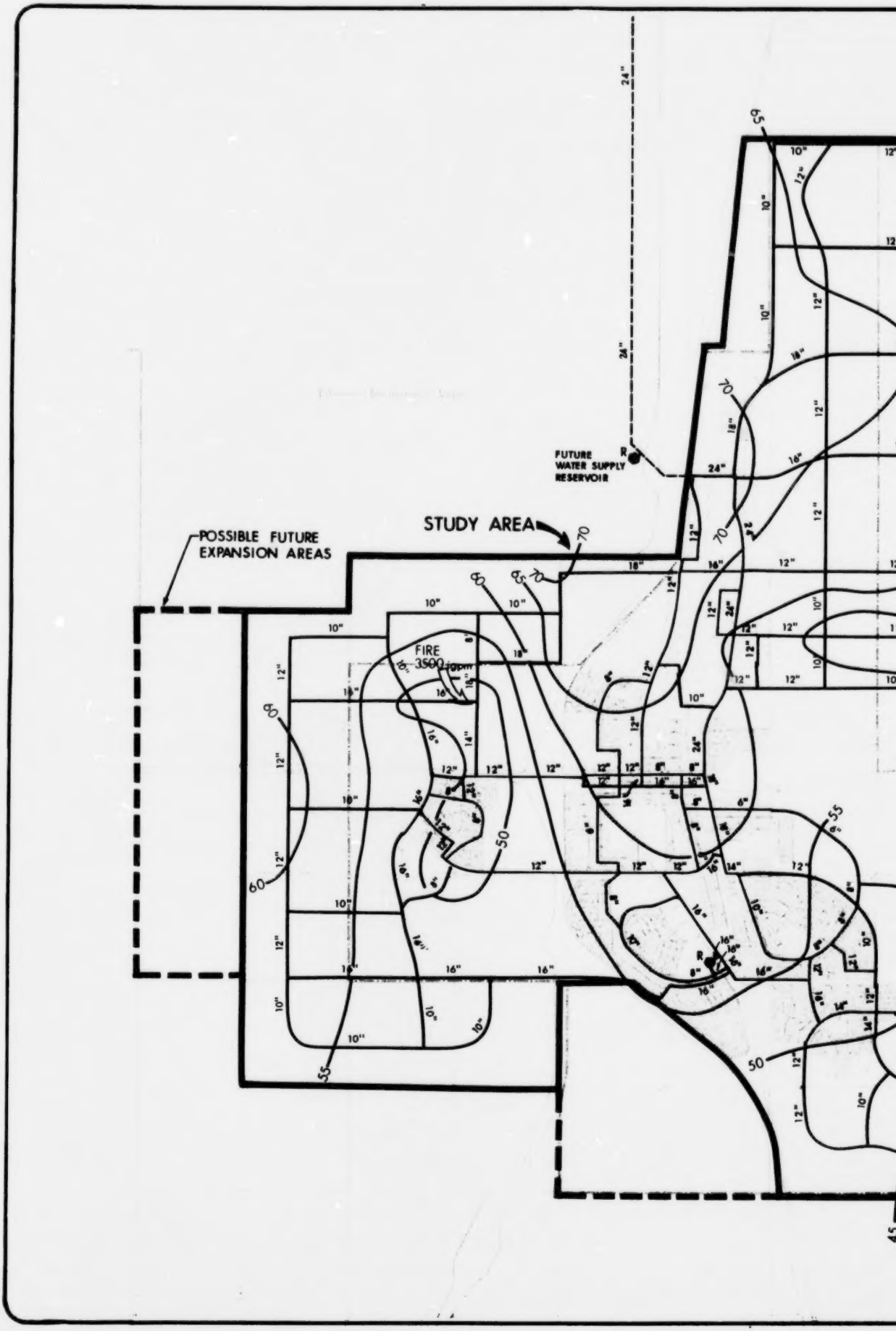
WATER FEEDER SYSTEM

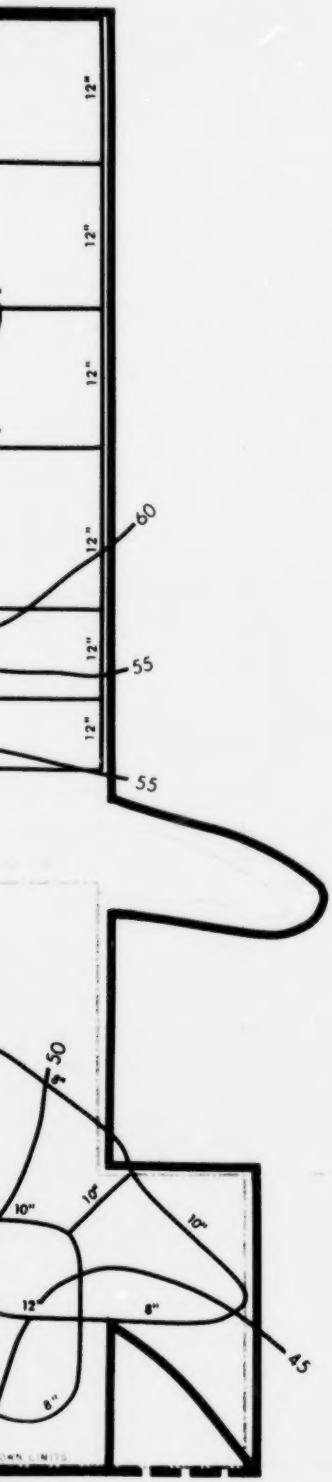
PEAK HOURLY DEMAND

TOWN OF LEDUC GENERAL MUNICIPAL PLAN

WATERWORKS & SEWERAGE

MAP 4


FUTURE WATER SUPPLY RESERVOIR
R
STUDY AREA
POSSIBLE FUTURE EXPANSION AREAS
FIRE 3500 gpm
24"
12"
10"
16"
18"
14"
8"
6"
70
60
50
55



LEGEND

12" EXISTING TRUNK MAIN

EXISTING SUPPLY LINE

R EXISTING RESERVOIR

16" PROPOSED TRUNK MAIN

60 60 PRESSURE CONTOUR IN POUNDS PER SQUARE INCH

PIPE DIAMETERS:

SI UNITS mm	IMP UNITS inches
200	8
250	10
300	12
350	14
400	16
450	18
600	24

NOTE: ONLY WATERMAINS INCLUDED IN ANALYSIS ARE SHOWN.

PRESSURE CONTOURS:

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172	25
207	30
241	35
276	40
310	45
345	50
379	55
414	60
448	65
483	70
517	75

ASSOCIATED ENGINEERING SERVICES LTD AESL

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kilometers 0 .5 1 1.5 2

miles 0 .5 1 1.5

Edmonton Regional Planning Commission

WATER FEEDER SYSTEM

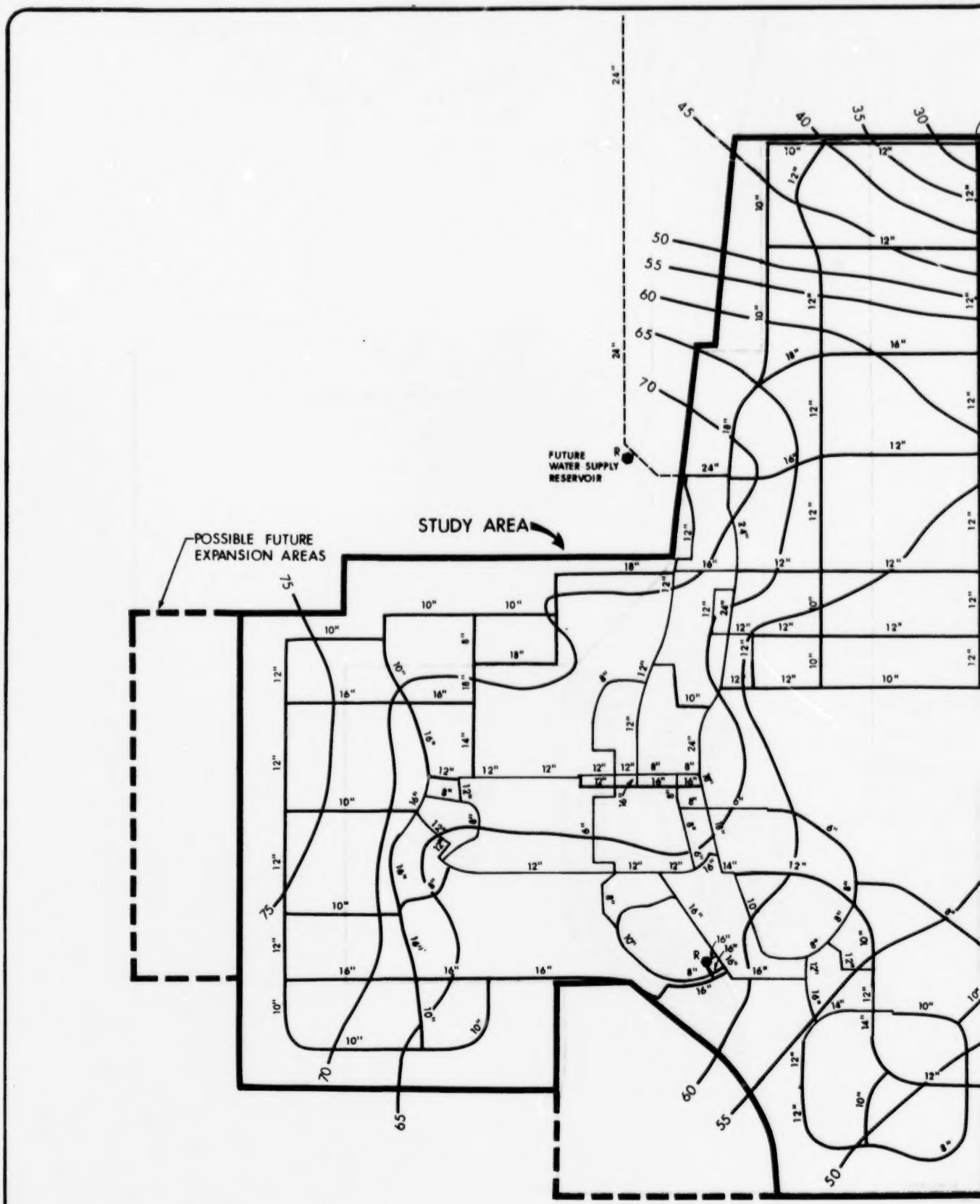
PEAK DAY DEMAND PLUS

COMMERCIAL FIRE FLOW

TOWN OF LEDUC GENERAL MUNICIPAL PLAN

WATERWORKS & SEWERAGE

MAP 5


FUTURE
WATER SUPPLY
RESERVOIR
R
STUDY AREA
POSSIBLE FUTURE
EXPANSION AREAS
45
40
35
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60
65
70
75

LEGEND

12" EXISTING TRUNK MAIN

EXISTING SUPPLY LINE

R EXISTING RESERVOIR

16" PROPOSED TRUNK MAIN

PRESSURE CONTOUR IN POUNDS PER SQUARE INCH

PIPE DIAMETERS:

SI UNITS mm	IMP UNITS inches
200	8
250	10
300	12
350	14
400	16
450	18
600	24

NOTE: ONLY WATERMAINS INCLUDED IN ANALYSIS ARE SHOWN.

PRESSURE CONTOURS:

KILOPASCALS	POUNDS PER SQUARE INCH
172	25
207	30
241	35
276	40
310	45
345	50
379	55
414	60
448	65
483	70
517	75

ASSOCIATED ENGINEERING SERVICES LTD AESL

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kilometers 0 .5 1 1.5 2

miles 0 .5 1 1.5

Edmonton Regional Planning Commission

WATER FEEDER SYSTEM

PEAK DAY DEMAND PLUS

INDUSTRIAL FIRE FLOW

TOWN OF LEDUC GENERAL MUNICIPAL PLAN

WATERWORKS & SEWERAGE

MAP 6

IV. SANITARY SEWERAGE SYSTEM

GENERAL

The sanitary sewerage system includes all facilities required to collect sewage from households, commercial and industrial establishments within the community, to transport the collected sewage to the sewage treatment works and to treat and dispose of the treated sewage.

A report prepared for Alberta Environment for the Southeast Regional Trunk Sewerage System, dated September 1979 considers various alternatives of sewage treatment and disposal for the region which includes Leduc. The accepted alternative provides for a regional trunk sewer to carry the sewage from Leduc and other areas in the region to a regional sewage treatment plant east of Edmonton with disposal of treated effluent in the North Saskatchewan River downstream from Edmonton.

The concept of the regional trunk sewer has been assumed in this study to apply to the treatment and disposal of sewage for the Town of Leduc. Only the required sizing, location and elevations of the proposed regional trunk sewer are reviewed and assessed as it relates to the ultimate requirements for Leduc.

The general slope of the terrain within the study area is from the southeast towards the northwest. Lift stations are required to transport sewage from several areas to the present treatment facilities in the east of the study area. The proposed regional trunk sewer, if constructed deep enough, will be able to eliminate some of the existing lift stations. A lift station will still be required for the western development areas.

Design criteria for the sanitary sewerage system are shown in Appendix B. The per capita sewage flow contributions of 591 L/c/d (130 igpcd) for the area between Highway #2 and the CPR main line reflects experiences in the Town with high amounts of extraneous flows and infiltration. For the other areas in the Town the more common contribution of 364 L/c/d (80 igpcd) has been used.

The measured sewage flows during 1979 varied from a minimum flow of approximately 136 L/c/d (30 igpcd) to a maximum flow of 536 L/c/d (118 igpcd). The average flow was 214 L/c/d (47 igpcd). These flows are composite domestic sewage flows, including commercial and industrial contributions, as determined from total sewage flows, divided by the actual Town's population.

Sewer mains must be sized to handle peak flows including reasonable infiltration and extraneous flows. Since infiltration and extraneous flows not only increase the required sizes of sewer mains but also effect the cost of sewage treatment, it is assumed that construction standards, control of sewer main installation and operation will be used to maintain infiltration and extraneous flows within allowable limits.

EXISTING FACILITIES

The existing sewage collection system brings the sewage through a 600 mm (24-in.) force main from the main lift station to the existing sewage treatment plant. The main lift station receives sewage from a 600 mm (24-in.) gravity sewer main from 46th Street north of 47th Avenue and a 250 mm (10-in.) force main along 47th Avenue, which conveys sewage from a lift station at 47th Avenue and 49th Street. The major portion of the town site between Highway #2 and the CPR main line discharges sewage into the 47th Avenue lift station through gravity sewers or from other lift stations.

The Qualico development east of the CPR main line is served by a 525 mm (21-in.) gravity trunk sewer which increases to a 600 mm (24-in.) sewer main at the point where a 400 mm (16-in.) force main from Leduc Estates enters into this trunk. The trunk sewer follows 46th Street to manhole 327 where it enters into the 600 mm (24-in.) gravity sewer to the main lift station.

Sewage from South Park is carried through a 375 mm (15-in.) trunk sewer discharging at Manhole 327. This trunk sewer also receives sewage from the lift station on 49th Street near Yonkers Avenue. This lift station collects the sewage from the trailer court east of Highway #2 and the industrial area to the east.

A lift station in the Weber Industrial Park also discharges into the 375 mm (15-in.) industrial gravity trunk sewer.

The major trunk sewers, collection mains, lift stations and force mains are shown on Map 7.

ASSESSMENT OF EXISTING FACILITIES

The existing collection system has been assessed on the basis of sewage contributions, set out in the design criteria of Appendix B. This represents a full development of the areas including redevelopment in the Central Business District. Under these conditions with high infiltration flows the system between Highway #2 and the CPR mainline will be overloaded at several points. The following items are noted:

1. Corinthia Park Trunk Sewer

The 300 mm (12-in.) Corinthia Park trunk sewer constructed through the Linsford Park and Central Area discharges into the 47th Avenue Lift Station. This trunk sewer becomes overloaded when subjected to peak flows and high simultaneous infiltration and extraneous flows.

2. 47th Avenue Lift Station and Force Main

The required capacity of this lift station is 179 L/s (6.30 cfs or 2,360 igpm). The present pumping capacity of the lift station is approximately 80 L/s (1050 igpm) requiring increased pumping capacity. The 250 mm (10-in.) force main under these conditions would be required to move sewage at a velocity of 3.66 m/s (12 fps) which is far in excess of a desired maximum velocity of 1.52 m/s (5 fps).

3. Willow Park Lift Station and Discharge

The capacity requirements for this lift station is 31 L/s (1.09 cfs or 408 igpm). Pumping capacity measured in 1978 was 15 L/s (200 igpm). The sewage is discharged through a 150 mm (6-in.) force main into the 200 mm (8-inch) gravity sewer on 50th Avenue. This gravity sewer also collects sewage from the Central Business District and cannot handle this load without surcharging.

4. Infiltration and Extraneous Flows

The extent of infiltration and extraneous flows into the sanitary sewage system is significant as shown in other reports (references 3 and 11) and as is witnessed during heav rainstorms. Both infiltration from ground water through weeping tile and extraneous flows into the sanitary sewers from streets and illegal connections or bypasses are the contributing factors. Limiting extraneous flows through continued efforts of finding the sources of extraneous flows will remain essential for the Town. Upgrading the storm drainage system will have beneficial effects. A separate third sewer system for weeping tile and ground water control is expensive and should only be considered as a last resort.

PROPOSED SYSTEM IMPROVEMENTS

Sewage lift stations allow development to take place in areas that otherwise cannot be serviced. They tend however to be continual problem facilities when required to handle the occasional overload or due to pump failure or power outage. Where at all possible, sewage lift stations should be eliminated for the more reliable gravity sewer mains.

The construction of the Regional Trunk Sewer offers the opportunity to eliminate several of the existing lift stations providing the proposed sewer is constructed deep enough.

It is suggested that the invert elevation of the proposed sewer at manhole 327 is approximately 720 m (2,362.2 ft.) and at Highway #2 and 65th Avenue 718 m (2,355.6 ft.). A shift of alignment is also recommended from that shown in the Alberta Environment report of September 1979.

With the suggested changes the following system improvements as shown on Map 7 can be accomplished:

1. Interceptor Sewer From 47th Avenue Lift Station to Regional Sewer

   Construction of a 600 mm (24-in.) interceptor sewer from the 47th Avenue lift station to the regional sewer at Manhole 327 will eliminate the lift station and provide for servicing the Alexander Park property slated for a civic centre development. Possible extension of this sewer south can provide for a relief of the Corinthia Park sewer.

2. Interceptor Sewer From 49th Street Lift Station to Regional Sewer

A short 250 mm (10-in.) interceptor sewer from the 49th Street lift station near Yonkers Avenue to the Regional sewer will eliminate this lift station.

3. West Trunk Sewer

An invert elevation of approximately 718 m (2,355.6) of the regional sewer at 65th Avenue will allow for a west trunk sewer serving the development areas west of Highway #2. The trunk sewer will not completely eliminate the need for a lift station in the west development areas. The Leduc Estates area will still be served by a lift station. The 400 mm (16-in.) force main will not be required. It is proposed that a permanent lift station be constructed in the northwest of the development areas. The ultimate capacity of this lift station is 354 L/s (12.46 cfs or 4,665 igpm) requiring a 500 mm (20-in.) force main. The force main discharges into a 750 mm (30-in.) branch of the west trunk sewer. The west trunk sewer is sized to serve the future development area between Highway #2 and Highway #2A and an additional 112.4 ha (278 ac) west of Highway #2 south of the study area boundary.

The lift station also serves the future development area at the west of the study area boundary.

4. Willow Park Lift Station

Construction of the west trunk sewer will allow the construction of a 250 mm (10-in.) gravity interceptor sewer from the Willow Park lift station to the west trunk sewer eliminating the lift station.

The interceptor will reduce the flow in the 50th Avenue sewer, postponing the need for increasing its size due to possible redevelopment in the Central Business District.

5. Weber Industrial Park

The lift station in the Weber Industrial Park can be eliminated by construction of an interceptor sewer directly into the Regional sewer.

6. North Industrial Development Areas

The industrial development areas in the northeast can be served by gravity sewers into the regional sewer along the north boundary of the study area.

7. Regional Sewer

In order to accommodate the above improvement and to serve the long term development of the Town the regional sewer should be 900 mm (36-in.) between Manhole 327 and 65th Avenue and 1,050 mm (42-in.) north of 65th Avenue. The total capacity of the trunk sewer is projected at 1,308 L/s (46.12 cfs) serving a population and equivalent population total of 153,559 persons. Special attention must be given to the proposed invert elevations of the regional sewer to assure that all proposed improvements can be connected. Since proposed elevations for the improvement were derived from a variety of plans and contour information, critical elevations must be confirmed by field survey.

COST ESTIMATES OF PROPOSED IMPROVEMENTS*

The cost estimates provided are based on conceptual design information only and set out the order of magnitude costs for the proposed improvements. Construction of facilities in some of the development areas may be a considerable time in the future and for this reason only cost estimates for the first phase of anticipated development are provided. The system improvement costs are summarized as follows:

1.	47th Avenue lift station interceptor 724 m (2,375 ft.) of 600 mm (24-in.) sewer	500,000.00
2.	49th Street lift station interceptor 170 m (558 ft.) of 250 mm (10-in.) sewer	38,000.00
3.	West trunk sewer from CPR branch line and Alton Drive to Regional Sewer 4,290 m (14,074 ft.) of 600, 675, 900 mm (24-27-36-in.) sewers	2,600,000.00
4.	Willow Park lift station interceptor 530 m (1,739 ft.) of 250 mm (10-in.) sewer	165,000.00
5.	Weber Park lift statin interceptor 335 m (1,100 ft.) of 375 mm (15-in.) sewer	120,000.00
6.	North Industrial development areas 7,800 m (25,590 ft.) of 375, 450, 525, 600 mm (15-18-21-24-in.) sewers	2,560,000.00
	TOTAL	5,983,000.00

* The estimated Costs are in 1981 dollars and include an allowance for engineering and contingencies.

V. STORM DRAINAGE SYSTEM

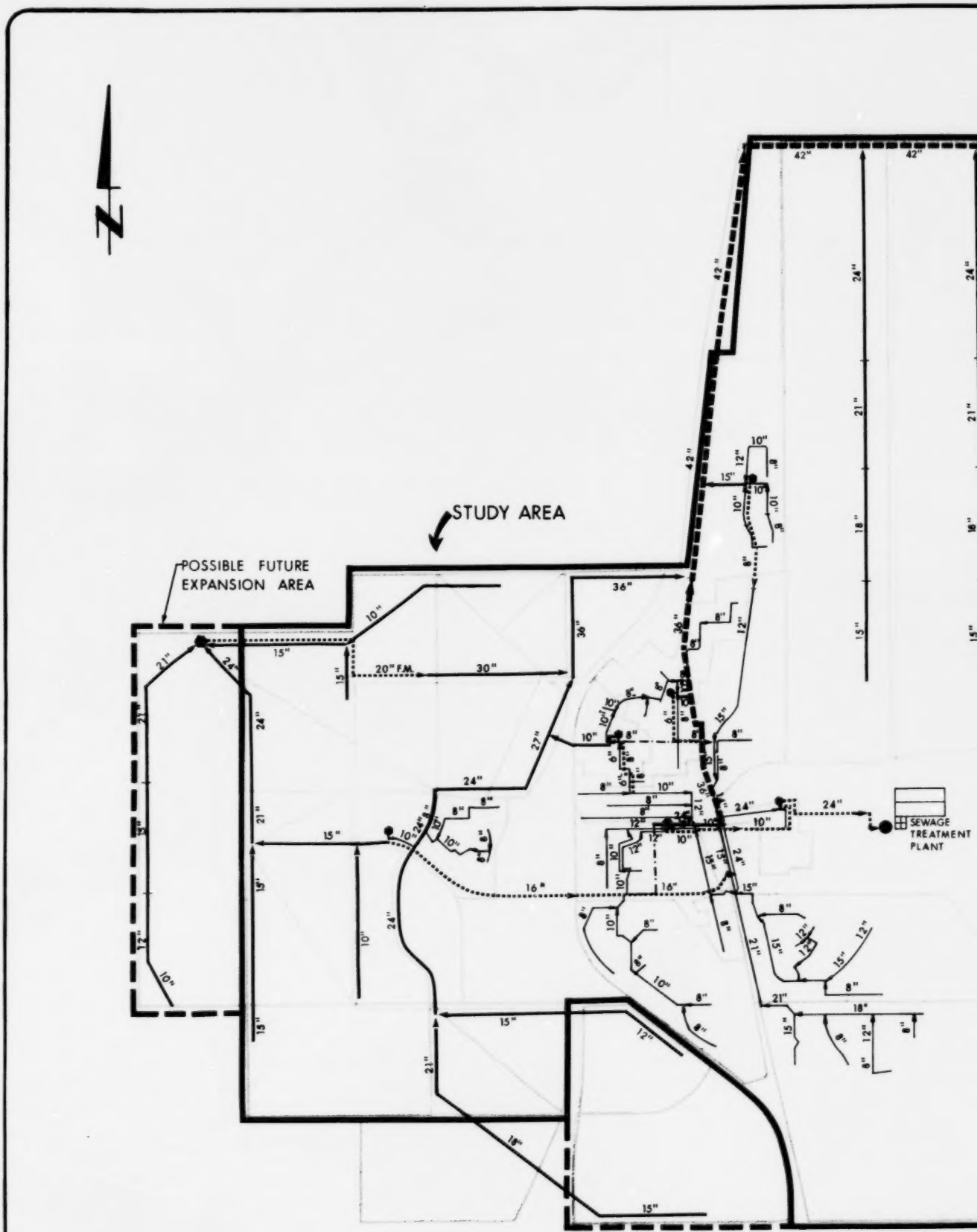
GENERAL

The major hydrologic changes resulting from urbanization are changes in the surface characteristics, changes in the natural drainage patterns and the channelization of natural surface flow into underground conduits. The major physical change in the surface characteristics resulting from urbanization is the increase of impervious areas.

The effect of changing the surface characteristics and drainage patterns due to urbanization is to alter the precipitation-runoff relationship. The extent of surface water infiltration is reduced and very little water percolates into the soil. Because of urbanization, ponding in streets and other developed areas cannot be tolerated and it is necessary to transfer the storm and surface water as quickly as possible to a receiving body or natural water course.

The net effect is that more water runs off relatively soon after a storm and this larger volume of water reaches the water course faster than would have occurred in predevelopment conditions.

The function of the storm drainage system is to transfer surface water runoff to natural water courses and water bodies in a manner which minimizes disruption to land uses and to the environment.


N
STUDY AREA
POSSIBLE FUTURE EXPANSION AREA
SEWAGE TREATMENT PLANT
20" F.M.

LEGEND

- EXISTING SEWER MAIN
- EXISTING FORCE MAIN
- EXISTING LIFT STATION
- PROPOSED SEWER MAIN
- PROPOSED FORCE MAIN
- PROPOSED LIFT STATION
- PROPOSED REGIONAL SEWER
- ALTERNATE RELIEF SEWER
- TRIBUTARY AREA BOUNDARY

SI UNITS mm	IMP UNITS inches
150	6
200	8
250	10
300	12
350	14
375	15
400	16
525	21
600	24
900	36

ASSOCIATED ENGINEERING SERVICES LTD AESL

N

kilometers 0 .5 1 1.5 2

miles 0 .5 1 1.5

Edmonton Regional Planning Commission

SANITARY SEWERAGE SYSTEM

TOWN OF LEDUC GENERAL MUNICIPAL PLAN

WATERWORKS & SEWERAGE

MAP 7

This report is to serve as a "Master Plan" for storm drainage to assist in development of the Town of Leduc.

TOPOGRAPHY

The developed portions of Leduc are concentrated along the Canadian Pacific Railway running generally north and south just west of Telford Lake. The overland trend in this area is gently sloping northward at an approximate slope of .25%.

Those areas expected to develop to the west of the existing Town core slope gently in a northwesterly direction to minor tributaries of Whitemud Creek. The slopes encountered in these areas are typically 0.4%.

The areas of future development north of the existing core and east of the CPR slope gently eastward to minor tributaries flowing into Saunders Lake. The existing topography slopes at grades between .2% to .4%.

Finally, developing areas south of the existing Town and east of the CPR slope naturally in a northward direction to Telford Lake at grades in the order of 0.60%.

EXISTING DRAINAGE SYSTEMS

The storm runoff from the Town of Leduc is presently handled by four separate drainage systems:

1. Corinthia Park Subdivision discharges into the existing "reservoir" west of Highway No. 2. Overflow from this reservoir enters a minor tributary of the Whitemud Creek.

2. Areas north of Corinthia Park, bounded by Highway No. 2 to the West and the Canadian Pacific Railway to the East, discharges into a ditch along the north boundary and then into a minor tributary of the Whitemud Creek, situated to the Northwest of the Town's existing boundary.

3. Part of the central area and areas east of the Canadian Pacific Railway discharges into Telford Lake.

4. The major portion of Leduc Estates is routed westward and discharges into a minor tributary of Whitemud Creek.

The existing drainage facilities include a combination of conventional storm sewer systems, open ditches and to a lesser extent, the utilization of the old highway and railway ditches as "trunk" drainage courses.

The water level of Telford Lake is subject to control by Ducks Unlimited. The Ducks Unlimited project controls the elevation of the Lake at an elevation of 726 m (2,382.0 ft.) above mean sea level. This is an elevation equivalent to the culvert inverts under the Canadian Pacific Railway.

This relatively high water elevation in Telford Lake could cause backup in storm sewers and flooding of downtown areas west of CPR during major storms and spring runoff periods. Regulation of runoff into Telford Lake or lowering the lake water level would reduce the possibility of potential flooding of developed areas.

EVALUATION OF EXISTING DRAINAGE FACILITIES

Historically, there have been problems with the disposal of storm water runoff in the Town of Leduc. Most recently, a severe rainfall event (approximately a 1:25 year storm) occurring on August 27, 1980 resulted in significant flood damage to property and caused serious traffic problems in the vicinity of 50th Street and north of 51st Avenue.

Although a storm sewer system was gradually developed in the present Town core, the 50th Street trunk sewer system was not designed to accommodate flows resulting from increased development in the Central Business District. To provide an acceptable level of protection to this area and to accommodate flows anticipated as a result of increased density and imperviousness in the downtown area, the capacity of the existing storm sewer system serving this area must be reassessed.

The minor tributaries of Whitemud Creek are poorly defined channels that meander freely across the countryside before eventually joining Whitemud Creek. Some sections of these tributaries pond water due to the lack of a continuous "positive" grade. To discharge storm water to these tributaries within the Study Area the discharge rate must be controlled to the predevelopment rate of runoff and the grade and channel sections of these tributaries must be improved to prevent flooding within these tributaries.

PROPOSED STORM DRAINAGE ALTERNATIVES

General

The basic design concept used in this report is to size the proposed drainage facilities to serve their tributary areas at ultimate development and to control the rate of runoff from the development area using the stormwater management approach. This approach includes runoff control by creating storage lakes, so the increased runoff from the developed area will not exceed the predevelopment or agricultural runoff. This approach makes the best use of the natural features of the land and utilizes the existing stream network in the final drainage system where possible.

A discussion of alternative drainage concepts and the stormwater management approach are included in Appendix D.

The proposed drainage system is shown on Map 8. The locations and configurations of the ponds shown are conceptual only and have been selected in accordance with natural topographic features, planning objectives and hydrologic effectiveness. Catchment areas for the individual ponds (storage facilities) and trunk sewers (minor systems) have been selected to minimize large diameter trunk sewers and depths of cut for installation while maintaining the Town's existing minimum standards regarding grades and cover over trunk sewers.

A conceptual design of the trunk sewer system (alignments and sizes) are also shown on Map 8. The alignments and sizes of trunk sewers must be reviewed and revised as required prior to construction.

Improvements to Existing Drainage Facilities

Improvements to the channel cross sections and grades of the Whitemud Creek tributaries will be required as development progresses. Since most of these tributaries originate outside the study boundary, a detailed hydrologic and hydraulic design should be carried out to ensure proper design and sizing of channel improvements. This type of analysis may be eligible for Provincial funding under Position Paper No. 5.

Solution of the drainage problems in the established Town core is a complex matter. Several alternatives for improving the drainage in this area may be considered.

Use of on-site controls in the form of roof top and parking lot storage may have a significant effect on peak flow reduction and reducing flood drainage. The disadvantage of on-site controls is that they require conversion of existing structures and parking facilities and require monitoring of new facility construction to ensure that they are correctly implemented and maintained.

A stormwater management facility may be provided in the form of a wet or dry storage pond to reduce peak flows and control high runoff volumes. However, since the existing Alexander Park area is near the upstream end of the tributary area, the effectiveness of this form of storage will be minimal.

A final alternative in alleviating the drainage problem in the Town core is to install a second trunk sewer along 50th Street to remove storm runoff from the developed area quickly and with a minimum of inconvenience. This alternative would be a major expense, pavement and curb structures on 50th Street would be disrupted. However, the construction of a relief

sewer based on a 1:10 year storm event and ultimate development condition would provide a satisfactory solution to the existing drainage problem in the established Town core. A relief storm sewer based on a 1:5 year storm event was analyzed and provided in addition to the existing system as shown on Map 8.

Implementation of any of these alternatives would greatly alleviate the existing drainage problems in the present Town. The most effective alternative is the relief storm sewer concept in parallel with the existing storm sewer along the old highway (50th Street).

Another possible solution of the existing drainage problem is to construct "interceptor" sewers to pick up flows at intermediate points on 50th Street and divert them into Telford Lake and to the outlet at Highway No. 2. This concept will involve extensive construction of large diameter sewers and the effectiveness of the solution is limited by the relatively high water level maintained in Telford Lake.

Drainage to Saunders Lake

It is proposed that drainage of the lands north of Telford Lake and east of the CPR (zoned as industrial) be drained eastward to Saunders Lake. Runoff from Sections 1-50-26-4, 12-49-26-4 and N 1/2 36-49-25-4 will be regulated with three separate retention ponds to predevelopment discharge rates. The predevelopment discharge will continue naturally eastward via existing channels to Saunders Lake.

Drainage to Telford Lake

The present drainage trend in Sections 24 and 25 of 49-25-4 and 19-49-24-4 is northward into Telford Lake. With the development of W-1/2 19-49-24-4, a parkway will provide an efficient drainage outlet to Telford Lake. The remaining developing areas south of Telford Lake and east of the CPR will drain to retention ponds to attenuate peak flows prior to discharging to the existing storm sewer for final discharge into Telford Lake.

Drainage to Northern Whitemud Creek Tributary

The bulk of the existing Town core east of Highway No. 2 and north of 44th Avenue eventually outlets storm runoff to this tributary in NW 35-49-25-4. A retention pond is proposed to attenuate the high peak flows anticipated from the shopping centre proposed for the NW 1/4 35-49-25-4 and NE 1/4 34-49-25-4. This pond will be oversized to control peak discharges from a portion of the existing Town into the tributary as regulation of flows through the Edmonton International Airport is critical. The effluent from this pond will be diverted westward by a proposed drainage parkway away from the Airport to the existing tributary of Whitemud Creek.

Drainge to Central Whitemud Creek Tributary

Existing sections of the Town of Leduc bounded on Highway No. 2 on the west, the CPR to the east and the south by 44th Avenue drain to this tributary via the existing reservoir west of Highway No. 2. Leduc Estates is drained westward to the existing municipal boundary and then is routed northward to the same tributary.

Developing areas along the length of this key tributary will contribute runoff, after storm peaks are attenuated through the use of storm retention facilities. It is proposed that the tributary be upgraded to a continuous parkway system as development progresses. Lands scheduled for development along this tributary include:
most of 23-49-25-4, NE 1/4 23-49-25-4, S 1/2 27-49-25-4,
most of 33-49-25-4, and 34-49-25-4.

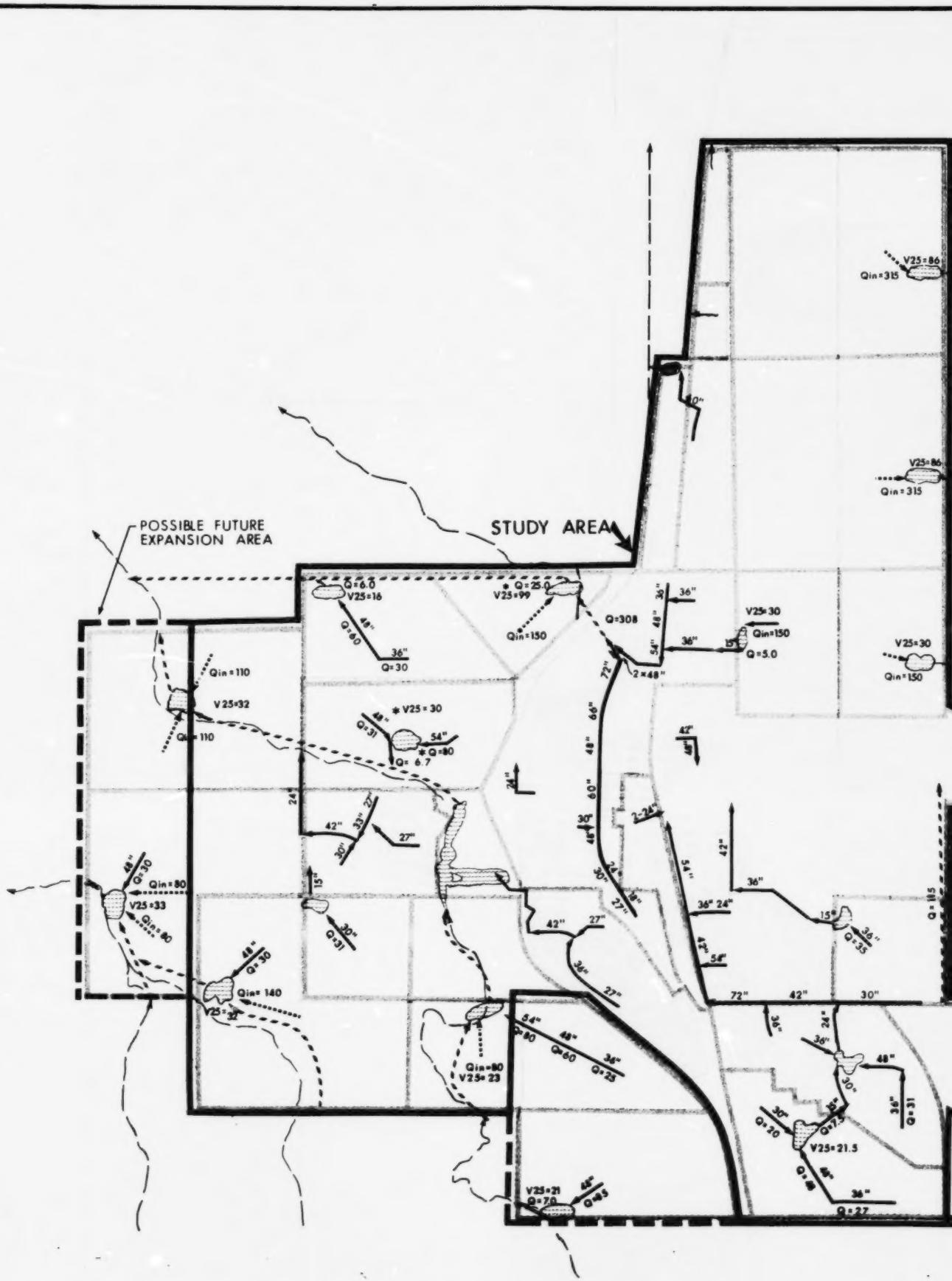
Drainage to Southern Whitemud Creek Tributary

It is proposed that the southern Whitemud Creek Tributary be upgraded to a parkway system also to serve NW 1/4 22-49-25-4, NE 1/4 21-49-25-4 and most of 26-49-25-4. Peak flow attenuation will again be achieved through the use of storm water retention facilities.

ESTIMATED COST OF PROPOSED IMPROVEMENTS

Estimated costs of the proposed drainage facilities shown on Map 8 to provide relief for the Town core from a 1:5 year rainfall event are estimated to be in the order of $2,320,000. These costs are in 1981 dollars and include an allowance for engineering and contingencies. These costs are provided for scheduling and budget purposes only and should be reviewed and updated as required prior to implementation.

The development of proposed retention ponds and the associated trunk and lateral sewers should be an integral part of the subdivision and land development process. Therefore, the costs of these facilities have not been included.


POSSIBLE FUTURE
EXPANSION AREA
STUDY AREA
V25=86
Qin=315
V25=86
Qin=315
Q=6.0
V25=16
48"
Q=60
36"
Q=30
* Q=25.0
V25=99
Qin=150
Q=308
36"
36"
V25=30
Qin=150
54"
48"
36"
Q=5.0
V25=30
Qin=150
72"
2-48"
Qin=110
V25=32
Qin=110
* V25=30
48"
Q=31
54"
* Q=80
Q= 6.7
42"
48"
24"
24"
42"
33"
27"
30"
27"
48"
60"
36"
2-24"
42"
48"
Q=30
Qin=80
V25=33
Qin=40
15"
30"
Q=31
42"
27"
54"
36"
36" 24"
42"
54"
15"
36"
Q=35
Q=185
48"
Q=30
Qin=140
V25=32
36"
27"
72"
42"
30"
Qin=80
V25=23
54"
Q=80
48"
Q=60
36"
Q=25
36"
24"
36"
48"
30"
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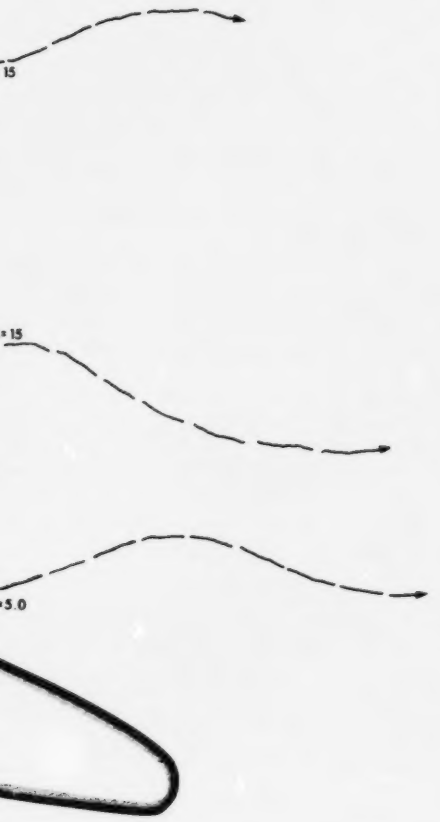
45"
Q=8
36"
Q=27



LEGEND

- EXISTING STORM SEWER
- EXISTING DRAINAGE CHANNEL
- EXISTING DETENTION POND
- EXISTING RETENTION POND
- PROPOSED STORM SEWER
- PROPOSED DRAINAGE PARKWAY
- PROPOSED RETENTION POND
- LOCAL INFLOW
- SUBCATCHMENT AREA BOUNDARY

V25=248 TWENTY FIVE YEAR VOLUME IN ACRE FEET (To convert to cubic metre multiply by 1233.0)

Q=30 FIVE YEAR FLOW IN CUBIC FEET PER SECOND (To convert to cubic metres per second multiply by 0.0283)

* CALCULATIONS BASED ON SHOPPING CENTRE DEV.

SI UNITS mm	IMP UNITS inches
375	15
450	18
600	24
675	27
750	30
900	36
1050	42
1200	48
1350	54
1800	72

NOTE: EXISTING CHANNELS TO BE UPGRADED AS REQUIRED

ASSOCIATED ENGINEERING SERVICES LTD. AESL

N

kilometers 0 .5 1 1.5 2

miles 0 .5 1 1.5

Edmonton Regional Planning Commission

STORM WATER MANAGEMENT SYSTEM

TOWN OF LEDUC GENERAL MUNICIPAL PLAN

WATERWORKS & SEWERAGE

MAP 8

VI. SUMMARY AND CONCLUSIONS

The population of the Town of Leduc increased from 7,500 in 1974 to 11,000 in 1979. To accommodate this rapid population growth and meet industrial land demands the Town made an arrangement with Edmonton Regional Planning Commission to update the General Municipal Plan and authorized Associated Engineering Services Ltd. to prepare servicing master plans for waterworks and sewerage systems within the Study Area of the General Municipal Plan.

This report evaluated and established the required waterworks and sewerage facilities for the General Municipal Plan. It should be emphasized that the proposed facilities are conceptual only and detailed analyses of these facilities are required before they are implemented.

Based on the study results the regional water supply facilities are adequate to meet the Town's existing and future water demands. Due to changes in operating philosophy of the regional water supply system the Town will be required to provide reservoir and repumping facilities at the supply point (near the master meter vault) to the Town. These facilities should be designed to meet the Town's 1991 projected peak day water demands.

The existing water distribution system is adequate to meet existing water demands. The existing system can be extended to service future development areas as shown on Map 4 to 6. As indicated by the pressure contours the proposed system will provide water distribution at adequate pressure throughout the system and during the various design periods.

Although the capacity of the existing distribution reservoir and repumping facilities in Corinthia Park meet the existing water demands, to provide adequate storage and service pumping capacity to year 1991 an additional 6,819 m^3 (1.5 mig) reservoir cell and one 102 L/s (1,350 igpm) service pump installation will be required in 1981.

The results of the study show that the capacity of several existing sanitary trunk sewers and lift stations are reached or exceeded particularly during wet weather conditions. Furthermore, Alberta Environment is studying an alternative mode of sewage disposal by which sewage from the participating communities would be collected by a regional sewer line and treated by a common (regional) sewage treatment facility on the bank of the North Saskatchewan River northeast of Edmonton.

Therefore, to improve sanitary sewer services within the existing developed areas and provide services in future subdivisions the following improvements should be considered:

1. The upgrading of existing sanitary sewerage system (trunk sewers and lift stations) between Highway 2 and CPR.

2. The implementation of the regional sewer line.

3. The implementation of the first stage of the west sanitary trunk sewer.

Based on the analysis of the drainage system, it was concluded that the conventional storm sewer system based on a 1 in 5 years design storm provides the least protection against flooding due to a rare storm event, such as a storm once in 25 years or once in 100 years.

To provide a high degree of protection against flooding and an adequate runoff control into existing lakes and streams in this report the Storm Water Management Approach was applied in the preliminary design of proposed drainage facilities. In accordance with the Terms of Reference, detailed hydrologic analysis of existing stream courses was not included in this report, but such analysis is imperative to alleviate future flooding in built-up areas due to potential backwater effect from existing lakes and stream courses. Recent flooding between the Highway and railway underlines the need for such study.

Recommendations for drainage improvements in the central business area were made. The development of proposed drainage facilities in other development areas should be an integral part of those developments and should be carried out in conjunction with the land development process.

Other improvements outlined in this study and local servicing of development areas must follow to keep pace with growth and expansion.

It is intended that this report will serve as a guide in the development process and will assist the Town in continuing on with a planned program of growth so that the waterworks and sewerage facilities provided today are beneficial tomorrow.

Respectfully submitted,

S. Vamosi, P.Eng.

APPENDIX A

DESIGN CRITERIA FOR WATERWORKS

APPENDIX A

DESIGN CRITERIA FOR WATERWORKS

WATER DEMANDS

The quantity of water consumed by the community is expressed at the average annual rate of use in litres per capita per day (gallons per capita per day). Two characteristic demand periods are recognized as being a critical factor in the design and operation of a waterworks system, namely the peak day and peak hourly demands.

PEAK DEMAND TRENDS

To provide adequate services today, waterworks systems must supply customers with all the water they want, whenever and wherever they want it. As most water utilities must be capable of meeting the maximum or near maximum demands, the estimation of peak demands is necessary for sound planning or improvements to the Town's water distribution system.

Based on previously established records, peaking water demands used in this report are as follows:

Peak Day =	2.0 x average day
Peak 5 Day Week =	1.7 x average day
Peak Hour =	3.0 x average day

EXISTING WATER CONSUMPTION

The Town's 1979 estimated average daily composite per capita water consumption was 218 L (48 imp gal). This water consumption is based on the 1979 population figure of 11,016 and 878,000 m^3 (193.1 mig) of estimated annual water demands.

PROJECTED FUTURE WATER CONSUMPTION

Based on the Town's water records and the water consumption pattern of similar municipalities in the Edmonton region, future water consumption in the Town is expected to increase as future residential and industrial land development occurs. Therefore, in the analysis and preliminary design of the waterworks system the following average daily water demands were used:

1.	Residential	296 L/c/d (65 igpcd)
2.	Commercial	16,851 L/ha/d (1500 igpad)
3.	Industrial	22,468 L/ha/d (2000 igpad)

Water demands used in this report for the preliminary design of waterworks facilities and network analysis of the distribution system is shown in Table A-1.

FIRE FLOWS

Fire flow rates and duration of fire flows are in accordance with the 1977 Guide to Recommended Practice for Water Supply and Public Fire Protection of the Insurance Bureau of Canada.

The fire flow rates and duration used in this report are:

Commercial Area	265 L/s (3500 igpm)
Industrial Area	227 L/s (3000 igpm)
Residential Area	114 L/s (1500 igpm)

Duration in hours:

Fire flow required:	less than 151 L/s (2000 igpm)	2.0 hours
	227 L/s (3000 igpm)	3.1 hours
	265 L/s (3500 igpm)	3.7 hours

WATER STORAGE REQUIREMENTS

Water storage facilities are required in a municipal waterworks system for the following reasons:

1. Water supply supplement during water supply interruption at the water source or in the regional water supply system.

2. Operational and economical aids. Operational aids consist of meeting peak hourly supplement from the distribution reservoir where surplus water is stored during low demand periods and used during high consumption period of the day. This type of operation avoids duplication and oversizing of distribution mains and provide flexibility of distribution system operation.

3. Emergency reserve against water supply interruption during servicing water supply pumping equipment.

4. Fire protection.

Storage capacity requirements are usually governed by the development of the existing water distribution system, reliability factors, topography, low pressure tolerance and economics.

Storage for operational aid to meet peak hourly demands is often economical because this will allow the supply source to be operated at a uniform rate during the 24 hours of use on a week day. Thus water demand at rates above the peak day water consumption is satisfied from storage. Approximately 25% of the peak day demand is considered to be a good average for peak demand storage and was one factor used to determine the total required storage for the waterworks sytem.

It is common practice to provide emergency reserve at the water distribution reservoir against service interruption such as shutdown of supply facilities, and for fire fighting purposes.

In this report, 25% of one average day water demand was used to establish the storage required for service interruption. Storage requirements for fire fighting purposes are based on a 265 L/s (3,500 igpm) fire flow for a 3.7 hour duration.

Based on the above criteria, the required water storage for the various development stages of the Town is shown in Table A-2.

TABLE A-1

PROJECTED WATER DEMANDS

BASED ON POPULATION PROJECTION B

Description		Development Stage Existing	1991	2001	Ultimate
1. Projected Population					
a) Residential		11,602	24,900	37,300	76,677
b) Equiv. Commercial		600	1,235	1,850	3,804
c) Equiv. Industrial		6,810	29,880	44,760	67,923
TOTAL		19,012	56,015	83,910	148,404
2. Per Capita Water Consumption	L/c/d	296	296	296	296
	igpcd	65	65	65	65
3. Water Demands					
a) Average Day	m^3	5 618	16 552	24 795	43 852
	mig	1.236	3.641	5.454	9.646
	L/s	65	192	287	508
	igpm	858	2,528	3,788	6,699
b) Peak Day	m^3	11 236	33 104	49 589	87 704
	mig	2.472	7.282	10.908	19.293
	L/s	130	383	574	1,015
	igpm	1,716	5,057	7,575	13,398
c) Peak Hour	m^3	16 854	49 656	74 384	131 556
	mig	3.707	10.923	16.362	28.939
	L/s	195	575	861	1,523
	igpm	2,575	7,585	11,363	20,096

TABLE A-2
WATER STORAGE REQUIREMENTS

Description		Development Stage Existing	1991	2001	Ultimate
A. SUPPLY STORAGE					
1. Required storage for one average day	m^3	5 618	16 552	24 795	43 852
	mig	1.236	3.641	5.454	9.646
2. Recommended Total Storage	m^3	6 819	13 638	27 276	40 914
	mig	1.500	3.000	6.000	3.000
B. DISTRIBUTION STORAGE					
1. Required storage for:					
a) Operational Aid .25 x peak day	m^3	2 809	8 276	12 397	21 926
	mig	.618	1.820	2.727	4.822
b) Supply Interruption 0.25 x average day	m^3	1 404	4 138	6 199	10 963
	mig	.309	.910	1.364	2.412
c) Fire Flow	m^3	3 532	3 532	3 532	3 532
	mig	.777	.777	.777	.777
TOTAL	m^3	7 746	15 946	22 128	36 421
	mig	1.704	3.508	4.868	8.012
2. Recommended Total Storage	m^3	6 819	13 638	20 457	40 914
	mig	1.500	3.000	4.500	4.000
3. Existing Storage	m^3	7 001	--	--	--
	mig	1.540	--	--	--

APPENDIX B

DESIGN CRITERIA FOR SANITARY SEWERAGE SYSTEM

APPENDIX B

DESIGN CRITERIA FOR SANITARY SEWERAGE SYSTEM

SEWAGE CONTRIBUTION

The following sewage flow criteria were used in review of the sewerage system and for sizing of proposed sewer mains:

1. Average Daily Sewage Contribution:

(a)	Residential:	- new areas 364 L/c/d (80 igpcd) - developed areas 591 L/c/d (130 igpcd).
(b)	Commercial:	17974 L/ha/d (1600 igpd/ac) or 50 equivalent population per ha (20 equivalent population per ac).
(c)	Industrial:	22468 L/ha/d (2000 igpd/ac) or 60 equivalent population per ha (25 equivalent population per ac).

2. Infiltration

(a)	Residential:	Included in the per capita contribution.
(b)	Commercial:	Included in equivalent population contribution.
(c)	Industrial:	5617 L/ha/d (500 igpd/ac)

3. Peak Hourly Sewage Flow

$$\text{Average daily flow} \times 1 + \frac{14}{4 + P^{0.5}}$$

where P = population in 1,000's

Leduc has experienced considerable wet weather flows in the sanitary sewerage system. Reports on infiltration and extraneous flows have been prepared for the Town pointing to required action to reduce the flows. It is assumed that the Town will continue its efforts to reduce and limit extraneous flows and infiltration. For the existing developed area between Highway #2 and the CPR main line contributing flows include an allowance of 295 L/c/d (65 igcd) for this purpose. For the remaining developed areas of the Town and new development areas the allowance is 68 L/c/d (15 igcd) over the residential per capita flow of 295 L/c/d (65 igcd). Since the peaking factor is applied to the infiltration and the extraneous flow allowance the per ha (per ac) allowance is approximately 8400 L (750 ig) for a population density of 44 persons per ha (18 persons per ac).

4. Projected Sewage Flows

Sewage flows for the developed areas have been projected on the basis of present population figures and densities with slight modification as suggested by the ERPC. The Central Business District of 48 ha (118 ac) has been projected at the equivalent population density of 50 persons per ha (20 persons per ac).

Sewage flows from future residential areas have been projected using an overall population density of 44 persons per ha (18 persons per ac). This is higher than present population densities. It can however be assumed that with increased housing costs the long term trend will be to higher population densities.

The catchment area boundary of the west trunk sewer extends outside the study area boundary in the southwest and an additional 112 ha (278 ac) has been included to calculate total contributing flows for the west trunk sewer.

Overall sewage contributions have been summarized in Table B-1.

TABLE B-1
SUMMARY OF SEWAGE CONTRIBUTIONS INTO REGIONAL SEWER

CONTRIBUTING AREA	Population and Equivalent Population CONTRIBUTION AREA	Population and Equivalent Population ACCUMULATIVE AREA	PEAK FACTOR	AVERAGE FLOW L/s	AVERAGE FLOW cfs	TOTAL PEAK FLOW L/s	TOTAL PEAK FLOW cfs
1. At start of regional sewer (MH 327)	34,490	34,490	2.42	168	5.91	406	14.30
2. 49 St. lift-station	1,500	35,990	2.40	178	6.27	427	15.05
3. West trunk sewer including Willow Park liftstation	63,537	99,527	2.00	448	15.81	897	31.62
4. Existing industrial area	6,075	105,602	1.98	474	16.71	945	33.32*
5. West half industrial expansion area	26,627	132,229	1.90	586	20.66	1148	40.47*
6. East half industrial expansion area	21,440	153,669	1.85	676	23.84	1308	46.12*

* includes industrial infiltration allowance of 5617 L/ha/d (500 igpd/ac)

PIPE SIZING

Gravity Sewer Mains

Sizing of gravity sewer mains is based on the Manning Formula. Design criteria used are:

1.	Roughness coefficient	n = 0.013
2.	Minimum velocity when pipe flowing full	v = 0.61 m/s (2.0 fps)
3.	Minimum pipe size for industrial subdivision	250 mm (10-in.)

Pressure Mains

Sizing of pressure mains is based on the Hazen-Williams formula, using a C value of 120 for asbestos cement pipe.

APPENDIX C

DESIGN CRITERIA FOR STORM DRAINAGE SYSTEM

APPENDIX C

DESIGN CRITERIA FOR STORM DRAINAGE SYSTEM

STORM RETURN PERIOD

RAINFALL INTENSITY

DESIGN METHOD

- Quantities of Runoff
- Concentration Time
- Runoff Coefficient

CAPACITY OF CONDUITS

- Coefficient of Roughness
- Velocity of Flow

CULVERT DESIGN

STORMWATER RETENTION PONDS

- General
- Detention Ponds
- Retention Ponds
- Drainage Parkway

APPENDIX C - DESIGN CRITERIA FOR STORM DRAINAGE SYSTEM

STORM RETURN PERIOD

For the purpose of this study, the following storm return periods were used:

(a)	For proposed conduit sizing	5 years
(b)	For proposed retention ponds sizing:	
	- up to design high water level (HWL)	25 years
	- up to one foot below adjacent properties	100 years

RAINFALL INTENSITY

In the design of the storm trunk sewers, the most recent rainfall intensity duration curves for the City of Edmonton were used.

The rainfall intensity-duration curves are expressed for various return periods in the form:

$$i_a v = \frac{a}{(t_d + b)^c}$$

where $i_a v$ = average storm intensity at time t_d.

t_d = storm duration

a, b & c are constants

Based on the above formula, the intensity equation for 5, 25 and 100 year storms are as follows:

Return Period	a	b	c	Intensity Equation (inch/hr)
5	25.6	5	.762	$i = \frac{25.6}{(t_d + 5)0.762}$
25	42.6	6	.776	$i = \frac{42.6}{(t_d + 6).776}$
100	56.7	6	.780	$i = \frac{56.7}{(t_d + 6).780}$

DESIGN METHOD

For the preliminary design of drainage facilities the Rational Method of design was used. The specific design criteria used in the Rational Method of design are:

1. Quantities of Runoff

Quantites of runoff were determined by the rational formula:
Q = Ci A:
where:

Q = quantity of runoff in cubic feet per second (cfs)
C = coefficient of runoff
A = catchment area in acres, tributary to the point under design.
i = average intensity of rainfall, in inches per hour.

2. Concentration Time (tc)

The time of concentration was computed on the basis of overland travel time combined with the travel time in the trunk system.

3. Runoff Coefficient (c)

Runoff coefficients express the ratios of runoff to rate of rainfall. Based on the land uses of the areas considered, the following weighted runoff coefficients were used:

(a)	Agricultural Land and Parks	0.1
(b)	Park and Recreation	0.2
(c)	Residential Land	
	Single Family Residences	0.35
	Multi Family Residences	0.55
(d)	Commercial Areas	0.70
(e)	Central Business District	0.90
(f)	Industrial Areas	0.60

CAPACITY OF CONDUITS

Capacity of open channels and storm sewers were determined by using the Manning formula:

$$Q = A \frac{1.486}{n} R^{2/3} S^{1/2}$$

Where:

Q = discharge in cfs

A = cross sectional area of flow in sq ft

R = hydraulic radius in feet

S = slope in feet per foot

n = coefficient of roughness

1. Coefficient of Roughness

Type of Conduit	"n" Values
Open channels (earth)	.030
Concrete Pipe	.013
Corrugated Steel Pipe	.024

2. Velocity of Flow

Open channels and storm sewer pipe sizing were based on minimum and maximum velocities by which deposition or erosion in the conduit can be prevented. Minimum and maximum velocities used in the concept design are as follows:

	Minimum		Maximum	
	m/s	fps	m/s	fps
Open Channel	0.3	1.0	1.1	3.5
Storm Sewer Pipe	0.9	3.0	6.1	20.0

CULVERT DESIGN

Culverts considered here are circular pipes and pipe-arches, with uniform barrel cross-sections throughout.

There are two main types of culvert flows:

1. Inlet Control

In this case, the cross-sectional area of the barrel, the inlet configuration of geometry and the amount of headwater or ponding, are of primary importance.

2. Outlet Control

In this case, the tail water in the outlet channel and the slope, roughness and length of barrel also should be considered in addition to the cross-sectional area of the barrel, the inlet configuration and the amount of headwater at the inlet and the culvert.

Criteria for culvert design are as follows:

1. The culvert shall convey the 5 year runoff without static head at the inlet end of the culvert.

2. The culvert shall convey the 100 year runoff, utilizing the maximum static head available at the inlet end of the culvert.

3. The capacity of the culverts shall be determined for inlet and outlet control conditions and the lesser of the two values shall be used as the discharge capacity.

Capacity for metal pipe culverts may be determined by using the appropriate nomographs published in the "Handbook of Steel Drainage and Highway Construction Products".

Capacity of concrete pipe culverts may be determined by using the appropriate nomographs published in the "Design Data" of the American Concrete Pipe Association.

STORM WATER RETENTION PONDS

General

Based on location in the drainage system, storage facilities may be designed to be in-line or off-line, at upstream or downstream portions of the drainage system. Based on function, storage facilities may be designed for single or multi-purpose use, and may be "detention" (dry) or "retention" (wet) ponds.

Detention Ponds (Dry Ponds)

The term "detention pond" in this report means an earthen reservoir with side slopes having a maximum slope of 7:1, grassed surface areas and inlet and outlet structures. The Detention Pond would receive the discharge of the drainage conduits, (either natural stream, open channels or storm sewer, as applicable) and retain the differences between the amount of runoff and amount of effluent, until the effluent rate exceeds the runoff rate, after which time the pond is gradually drained.

Detention ponds are intended to maintain the effluent and storage in balance with the runoff due to a 48 hour storm having a frequency of occurrence of once in 25 years, and provide a minimum of 1.0 m (3.3 ft) free board over the 25 years water level rise to protect the area around the pond due to a 1 in 100 year storm.

Retention Ponds (Wet Ponds)

The term "Retention Ponds" in this report means an earthen reservoir with minimum side slopes of 7:1. The Retention Ponds were designed to temporarily store the difference in volume produced by a one-in-25 year storm of 48 hours duration for the contributing area, having runoff coefficients anticipated at ultimate development and the one-in-5 year storm, having an equivalent agricultural runoff rate.

The various storm water retention ponds were sized such that the normal minimum water depth would be approximately 2.4 m (8 ft) in order that the ponds could support fish life, if so desired. However, should the ponds not be required to support fish life, water depths (and hence construction costs) could be reduced. In this case, water depths could be reduced to approximately 1.5 m (5 ft) in order to maintain the water in a relatively fresh condition.

With normal water depths greater than 1.5 m (5 feet), it will be possible, in most cases, to keep inlet and outlet structures fully submerged and hence, hidden from view. These structures, therefore, will not detract from the natural appearance and aesthetics of the ponds.

Retention ponds were designed to allow a water level rise above the normal water level not more than 1.2 m (4 ft) for small ponds and not more than 1.5 m (5 ft) for larger ponds. To accommodate the runoff due to a storm once in 100 years, a water level rise of 2.4 m (8 ft) above the normal water level was established as a design guideline.

APPENDIX D

ALTERNATIVE DRAINAGE CONCEPTS

ALTERNATIVE DRAINAGE CONCEPTS

There are two basic concepts that can be considered in the design of storm drainage facilities for the Study Area. These two concepts and their various components are described below.

Conventional System

A "Conventional System" (commonly called the "minor system") consists primarily of a network of underground storm sewer pipes discharging to a receiving stream.

Such a system does have limitations in flood protection of properties where "emergency" overflow surface channels (the "major system") such as roadways, parkways, etc., are not provided to aid in directing runoff volumes from more severe rainstorms away from urban development.

Stormwater Management System

When rural areas are developed and urbanized, the result is that the volume of surface runoff and the rate of such runoff may exceed that of predevelopment by a factor of 2 to 5 times. The impact of such runoff increases can be significant and detrimental to downstream drainage facilities.

Stormwater Management is an approach to deal with the problem of increased runoff volumes and increased runoff rates from areas converted to urban development. In an attempt to manage stormwater, one primary objective is to "manage" or control the runoff such that the actual peak runoff rate for postdevelopment does not exceed the peak runoff rate of predevelopment. The basic mechanism for controlling the increased urban runoff is "storage". Storage permits "withholding" a significant portion of the runoff and allows for eventual discharge at reduced rates. A second primary objective is to recognize that any piped system has definite limitations, and as such, stormwater runoff from low lying areas of development must be "managed" and discharged out of such low areas through carefully preselected surface facilities (i.e. - roadways, drainage parkways).

Three components to be considered in a storm water management approach are as follows:

1. Major Drainage Facilities

   In any given urbanized and developed area, surface runoff is usually conducted through portions of and/or out of the area along roadways, overflow swales and drainage parkways in open space corridors. Such surface facilities are referred to and make up the "Major Drainage Facilities".

   Topography, development patterns, land availability, economics and general preference often limits the application of a complete major system as a means of providing storm drainage to any given area.

2. Minor Drainage Facilities

A system of preselected closed conduits, open conduits and related appurtenances (catch basins, outfall structures, etc.) is referred to and makes up the "Minor Drainage Facilities". Such facilities are designed to handle a finite amount of runoff - usually the amount of storm water expected as a result of a storm that occurs once in five years for residential areas and for a one in ten year storm in commercial areas.

3. Storage Facilities

The storage required for runoff control can be achieved by many small control facilities (building roofs, parking lots, landscaped depressions) or by larger centralized facilities (large urban retention ponds and/or detention ponds) in parks and open spaces.

Retention ponds are in fact, man-made water bodies that retain a preselected minimum amount of water year round in a natural lake type setting.

The advantages of retention ponds are:

(a) When designed to collect both minor and major system flows they provide effective runoff control for a wide range of storm frequencies and hence provide a higher level of flood protection than a conventional system.

(b) When adequately landscaped they can be developed as the focal point of the community and as part of an interconnected open park system with increased aesthetic and recreational values.

(c) Properties located adjacent to retention ponds will appreciate in value thereby increasing municipal tax revenue.

The disadvantage of retention ponds are:

(a) Effort and appropriate costs must be applied to "operate" and maintain the facility.

(b) In some cases, land otherwise available for development, is used for retention ponds.

(c) Some sectors of the public view them as a potential health and safety hazard.

The operation of retention ponds, as a means of storage for runoff control, is as follows:

(a) During dry weather periods the low pond water level is maintained, if necessary, by pumping on-site well water into the pond.

(b) During periods of runoff from the development area, the runoff enters the pond faster than the submerged outlet allows it to flow out, resulting in a gradual increase in the pond level.

(c) In the case of a very severe storm, (greater than a one in 100 year event) possible excess runoff from the pond overflows into the major system (swales in open space corridors) and is conveyed to other downstream retention ponds or directly to the receiving stream.

Detention storage facilities are, in most cases, man-made facilities intended to hold water runoff for a limited time only and as such are usually free of open water ponding for the majority of time.

The advantages of detention storage facilities are:

(a) When designed to collect both minor and major system flows, they provide effective runoff control for a wide range of storm frequencies, and hence provide a higher level of flood protection than a conventional system.

(b) When adequately landscaped they can be developed as an open-space, passive-park type of facility connected to adjacent open space areas.

The disadvantages of detention storage facilities are:

(a) Some effort and appropriate costs must be applied to maintain the facility (similar to maintaining an open space park).

(b) In some cases, land otherwise available for development, is used for detention storage facilities.

(c) Some sectors of the public view them as a potential health and safety hazard at those times that the facility ponds water.

The operation of detention storage facilities, as a means of storage for runoff control, is as follows:

(a) During summer dry weather periods, the facility is free of water and there are no operational requirements other than park maintenance.

(b) During periods of runoff from the development area, the runoff enters the facility faster than the outlet allows it to flow out, resulting in a water ponding condition for a short time.

(c) In case of a very severe storm (greater than a one in 100 year frequency) possible excess runoff from the facility overflows into the major system (swales in open space corridors) and is conveyed to other downstream runoff control facilities or directly to the receiving stream.

In this report, retention ponds were used in the stormwater management system because of their aesthetic and operational advantages over the detention or dry ponds. The 1:25 year storage volume and effluent rate for each retention pond are shown on Map 8, entitled "Storm Water Management System".